Principles of Bridge Reliability

application to design and assessment codes

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PRINCIPLES OF BRIDGE RELIABILITY - APPLICATION TO DESIGN AND ASSESSMENT CODES

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PRINCIPLES OF BRIDGE RELIABILITY - APPLICATION TO DESIGN AND ASSESSMENT CODES

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Summary

The paper gives a brief introduction to the basic principles of structural reliability theory and its application to bridge engineering. Fundamental concepts like failure probability and reliability index are introduced. Ultimate as well as serviceability limit states for bridges are formulated, and as an example the reliability profile and a sensitivity analysis for a corroded reinforced concrete bridge is shown.

Further the paper presents the development of new bridge design codes in the United States and Canada. They are based on a probability approach. Structural performance is measured in terms of the reliability index. The major steps include selection of representative structures, calculation of reliability for the selected bridges, selection of the target reliability index and calculation of load and resistance factors. Load and resistance factors are derived so that the reliability of bridges designed using the proposed provisions will be at the predefined target level.

Introduction

In the paper a brief introduction to modern structural reliability theory and its application to assessment of the reliability of bridges is presented. For the so-called fundamental problem with one stress effect variable and one load variable the failure probability concept is introduced. The fundamental case is then generalized to the general case with several correlated uncertainty variables.

To be able to assess the reliability of bridges, relevant limit states must be formulated. In the paper ultimate limit states (ULS) as well as serviceability limit states (SLS) are defined. As an example of the assessment of the reliability of a bridge a simple concrete slab bridge with corroded reinforcement is analysed. Three models for corrosion of reinforcement are formulated and the reliability profile is calculated. Further, the importance of die sensitivity analysis is emphasized.

The paper also presents the development of a new load and resistance factor design (LRFD) bridge code in the United States (ref. 1) and Canada (ref. 2). They are based on a probability approach. The major steps include selection of representative structures, calculation of reliability for the selected bridges, selection of the target reliability index and calculation of load and resistance factors. Load and resistance factors are derived so that the reliability of bridges designed using the proposed provisions will be at the predefined target level.

The load models are developed using the available statistical data, surveys and other observations. Load components are treated as random variables. Live load covers a range of forces produced by vehicles moving on the bridge. For multilane bridges, the maximum load effect is determined by simulations. The dynamic load is a function of three major parameters: road surface roughness, bridge dynamics (frequency of vibration) and vehicle dynamics (suspension system). The derivations are based on the numerical simulations.

The capacity of a bridge depends on the resistance of its components and connections. Structural performance is measured in terms of the reliability index. The reliability indices are calculated for girder bridges, including non-composite steel, composite steel, reinforced concrete and prestressed concrete girders. The results show a considerable degree of variation. The calculated reliability indices served as a basis for the selection of the target reliability index.

The Fundamental Reliability Problem

In some simple cases the reliability of a structure or a single structural element is determined by only two independent stochastic variables (a load effect variable S and a resistance variable R) and a single failure criterion $R - S < 0$. The failure probability P_f is then defined as the probability that $S > R$, i.e.

$$P_f = P(R - S \leq 0) \tag{1}$$

It is easy to show , see (ref. 3), that P_f can be written

$$P_f = \int_{-\infty}^{\infty} F_R(x) f_S(x) dx \tag{2}$$

where F_R, is the distribution function of R, and where f_S is the density function of S. This is illustrated in figure 1.

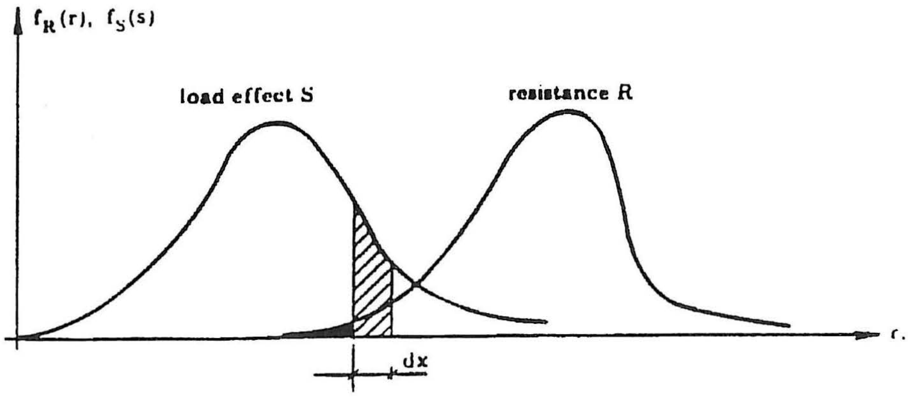

Figure 1. The fundamental reliability case.

The General Reliability Problem

In general, the reliability of structure is controlled by more than two stochastic variables. Let us assume that n stochastic variables X , $X_2, \ldots, X_n$ are needed to model the reliability of a structure. Then the joint density function for these variable must be defined and a failure function (limit function) $g(X_1,..,X_n)$ must be formulated in such a way that the structure is safe when $g(\cdot) > 0$ and in a failure state when $g(\cdot) \leq 0$. The failure probability is then defined by

$$P_f = P(g(x_1, \ldots, x_n) \leq 0) \tag{3}$$

Exact evaluation of this integral is in general not possible so some kind of approximation is needed. For this purpose the so-called reliability index β has been introduced (ref.3). This index is a measure of the reliability of the structure. It is in general relatively easy to calculate, and it can be shown that

$$P_f \approx \Phi(-\beta) \tag{4}$$

where Φ is the standard normal distribution function.

Limit States for Bridges

The following four limit states are usually selected for a reliability analysis of a concrete bridge:

- ultimate limit state (ULS):
  - collapse limit state (member or system)
  - shear failure limit state
- serviceability limit state (SLS):
  - crack width limit state
  - deflection limit state.

For steel bridges a fatigue limit state will in general also be needed.

Deterioration

In this section a brief presentation of modelling of chloride induced corrosion of reinforcement in concrete slabs/beams is given. The corrosion initiation period refers to the time during which the passivation of steel is destroyed and the reinforcement starts corroding actively. The rate of chloride penetration into concrete, as a function of depth from the concrete surface and time, can be represented by Fick's law of diffusion. The corrosion initiation period can then be calculated on the basis of the initial chloride content, the chloride content on the surface, the diffusion coefficient, the critical chloride concentration, and the cover. When corrosion has started then the diameter of the reinforcement bars at the time t is modelled by a linear function with time.

Reliability Profiles for Concrete Slab Bridges

A simply supported reinforced concrete bridge (see figure 2) is used for illustration of reliability assessment of a bridge. It is a short span bridge built in 1975 in the UK. The dimensions of the bridge and the reinforcement is shown in figure 2. The analysis is described in more detail in (ref.4).

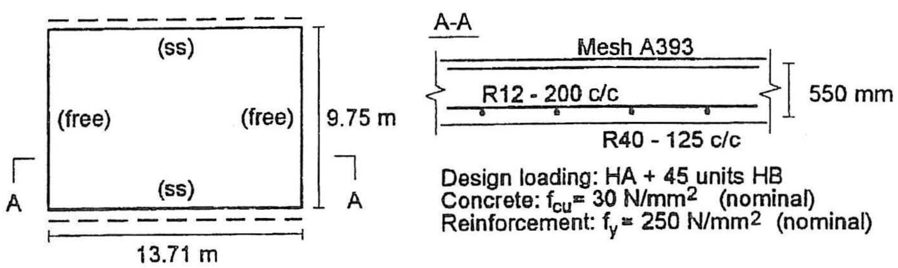

Figure 2. Bridge data.

For this particular bridge the corrosion model described above results in the corrosion development shown in figure 3. Figure 3 shows the reinforcement area as function of the time t in the interval 0 to 120 years.

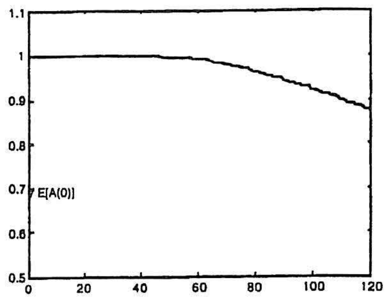

Figure 3. Reinforcement area $A(t))$ as a function of time.

The general highway traffic load model in the EUROCODE (ref. 5) for lane and axle load is applied. The load effects produced by the EUROCODE model are multiplied by a static stochastic load factor (extreme type 1) and a dynamic stochastic load factor (normal). Based on stochastic modelling of the bridge and reinforcement dimensions, the concrete and reinforcement strength properties, the loading and the corrosion the reliability profile for the collapse load limit state are shown in figure 4. The collapse is defined as formation of a yield line collapse mechanism.

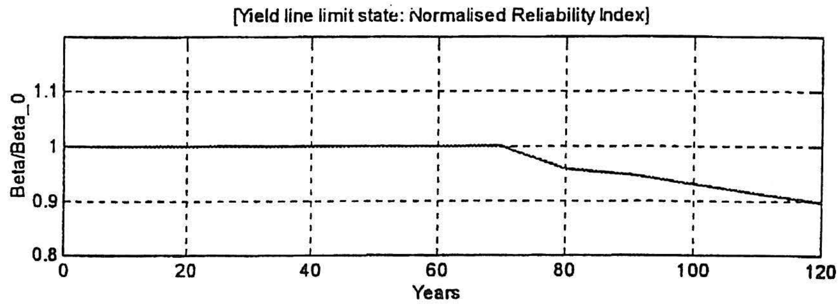

Figure 4. Reliability profile using a yield line limit state.

The results from a sensitivity study gives the change of the reliability index when changes in the stochastic variables parameters are made. For $t = 0$ year and $t = 120$ years the results of such sensitivity analyses are shown in figure 5.

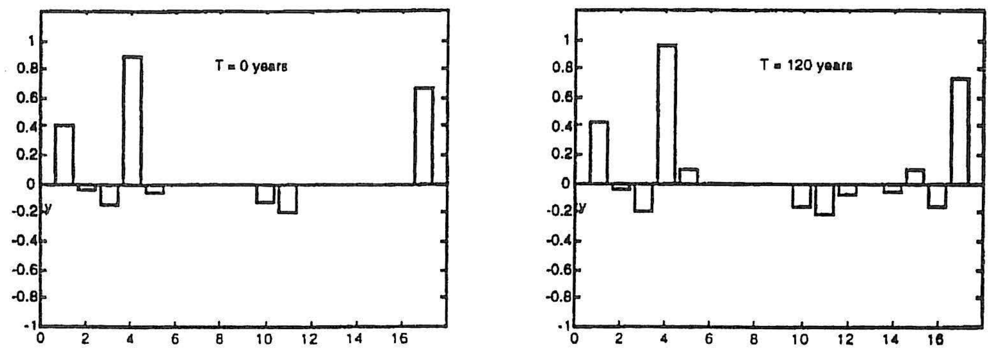

Figure 5. Sensitivity analysis for yield line limit state at $t = 0$ years and at $t = 120$ years.

Calibration Procedure

The calibration procedure was based on (refs. 6-7). The work on the new bridge design code was formulated including the following steps:

(a) Selection of representative bridges

Representative structures were selected from various geographical regions of the United States (AASHTO) and the Province of Ontario (OHBDC). These structures cover materials, types and spans which are characteristic for the region. Emphasis is placed on current and future trends, rather than very old bridges. For each selected bridge, load effects (moments, shears, tensions and compressions) are calculated for various components. Load-carrying capacities are also evaluated.

(b) Establishing the statistical data base for load and resistance parameters.

The available data on load components, including results of surveys and other measurements, are gathered. Truck survey and weigh-in-motion (WIM) data are used for modelling live load. There is little field data available for dynamic loads, therefore, a numerical procedure is developed for simulation of the dynamic bridge behaviour. Statistical data for resistance include material tests, component tests and field measurements. Numerical procedures are developed for simulation of the behaviour of large structural components and systems.

(c) Development of load and resistance models.

Loads and resistance are treated as random variables. Their variation is described by cumulative distribution functions (CDF) and correlations. For loads, the CDF's are derived using the available statistical data base (Step b). Live load model includes multiple presence of trucks in one lane and in adjacent lanes. Multilane reduction factors are calculated for wider bridges. Dynamic load is modelled for single trucks and two trucks side-by-side. Resistance models are developed for girder bridges. The variation of the ultimate strength is determined by simulations. System reliability methods are used to quantify the degree of redundancy.

(d) Development of the reliability analysis procedure.

Structural performance is measured in terms of the reliability, or probability of failure. Limit states are defined as mathematical formulas describing the state (safe or failure). Reliability is measured in terms of the reliability index, β. Reliability index is calculated using an iterative procedure. The developed load and resistance models (step c) are part of the reliability analysis procedure.

(e) Selection of the target reliability index.

Reliability indices are calculated for a wide spectrum of bridges designed according to the previous editions of AASHTO (ref.8) and OHBDC (ref.9). 'Me performance of existing bridges is evaluated to determine whether their reliability level is adequate. The target reliability index, β_T, is selected to provide a consistent and uniform safety margin for all structures.

(f) Calculation of load and resistance factors.

Load factors, g, are calculated so that the factored load has a predetermined probability of being exceeded. Resistance factors, ϕ, are calculated so that the structural reliability is close to the target value, β_T.

Load and Resistance Models

Load and resistance parameters are random variables. For steel girder bridges (non-composite and composite), reinforced concrete T-beams, and prestressed concrete girder bridges (pretensioned) the statistical models of resistance were developed in (ref. 10- 1 5).

It was determined, that the bias factor (ratio of mean to nominal) for dead load is λ = 1.031.05, and coefficient of variation V = 0.08-0.10. For live load, depending on span length, for AASHTO λ = 1.6-2.1, for OHBDC λ = 1.0-1.25, and V = 0.12. The nominal live load is represented by HS-20 truck (AASHTO 1992) and OHBDC truck (1983). HS20 loading consists of either three axles: 35 kN, 142 kN and 142 kN, spaced 4.3m, or a uniformly distributed lane load of 9.3kN/m with a moving concentrated force of 80kN. In the new LRFD AASHTO Code (ref. 1), live load is a combination of HS-20 truck and a uniformly distributed load of 9.3 kN/m. Therefore, the bias factor for live load is λ = 1.25-1.35. OHBDC (ref. 9) truck is a five axle vehicle: 60-140-140-200-160 kN, or a combination of 70% truck plus a uniformly distributed load of 10 kN/m. To make the bias factor more uniform, in OHBDC (ref. 2) the design truck has tandem axles increased to 160 kN each (instead of 140 kN). The corresponding bias factor is λ=0.95-1.10.

Dynamic load associated with an extreme value of truck load is about 0.10-0.15 of the static portion of live load, with V = 0.80. For a combined static and dynamic live load V = 0.18. design dynamic load in AASHTO LRFD (ref. 1) is specified as 33% of the truck load effect (with zero assigned to the uniform load). In OHBDC (ref. 2), dynamic load is assumed equal to 0.25 of static live load, except for very short spans governed by a single axle or a tandem.

The basic random variables considered in development of resistance models are dimensions, concrete compressive strength, and properties of structural steel, prestressing and non-prestressing strands. The parameters for moment carrying capacity are λ = 1.12 and V = 0.10, for non-composite and composite steel girders, λ = 1.14 and V = 0.13, for reinforced concrete T-beams, and λ =1.05 and V = 0.075, for prestressed concrete AASHTO-type girders. For shear capacity the parameters are λ = 1.14 and V = 0.105 for steel girders, λ = 1.20 and V = 0.155 for reinforced concrete T-beams, and λ = 1.15 and V = 0.14 for prestressed concrete AASHTO-type girders.

Reliability Analysis Procedure

Reliability indices, β, are calculated using a specially developed computer procedure based on the first order reliability method. The available reliability methods are reviewed in several textbooks (refs. 3 and 16). The methods vary with regard to accuracy, required input data, computational effort and special features (time-variance). In some cases, a considerable advantage can be gained by using the system reliability methods. The structure is considered as a system of components. In the traditional reliability analysis, the analysis is performed for individual components. Systems approach allows quantification of the redundancy and complexity of the structure. The new AASHTO LRFD (ref. 1) and OHBDC (ref. 2) codes are based on element reliability. However, system reliability methods are used to verify the selection of redundancy factors.

Structural performance is measured in terms of the reliability index β (ref.3).

Reliability Analysis For AASHTO (1992)

To develop a reference spectrum of the reliability indices, β, they were calculated for girders designed using the AASHTO (1992) and OHBDC (1983). In AASHTO (1992), the basic design requirement is expressed in terms of moments or shears (Load Factor Design),

$$1.3D + 2.17(L + I) < \phi R \tag{5}$$

where

D, L and I	are moments (or shears) due to dead load, live load and impact,
R	is the moment (or shear) carrying capacity, and
ϕ	is the resistance factor.

Values of the resistance factor are ϕ = 1.00 for moment and shear in steel girders, ϕ = 0.90 and 0.85 for moment and shear in reinforced concrete T-beams, respectively, ϕ = 1.00 and 0.90 for moment and shear in prestressed concrete AASHTO-type girders, respectively.

In OHBDC (1983), the basic design requirement is,

$$1.1D_1 + 1.2D_2 + 1.5D_3 + 1.4(L + I) < \phi R \tag{6}$$

where

D_1	is the dead load moment (or shear) due to factory-made components;
D_2	is the dead load moment due to cast-in-place components;
D_3	is the dead load moment due to asphalt;
L and I	are moments (or shears) due to live load and impact;
R	is the moment (or shear) carrying capacity, and
ϕ	is the resistance factor.

Values of the resistance factor are specified for material rather than components, and ϕ = 0.90 for moment and shear in steel, ϕ = 0.70 in concrete in composite steel girders; ϕ = 0.85 for steel rebars and prestressing steel, ϕ = 0.75 for shear capacity of rebars.

For AASHTO (1992), the results of calculations show a considerable variation in reliability indices depending on limit state and span length, from about 2 for short span (10m) and short girder spacing (1.2m) to over 4 for larger spans and girder spacing. The target reliability index was selected β_T = 3.5. For OI-IBDC (1983), reliability indices vary from 3 for short span (20m) to 4 for spans of 40-60 m, for steel girders and reinforced concrete T-beams. For prestressed concrete girders β is about 5. The same target reliability indeks, β_T =3.5, was selected.

New Load And Resistance Factors

The results of the reliability analysis for the current AASHTO (ref. 8) served as a basis for the development of more rational design criteria for the considered girders. The load factors developed for the LRFD AASHTO (ref. 1) are

$$1.25D + 1.50D_A + 1.75(L + I) < \phi R_n \tag{7}$$

where

D = dead load,

D_A = dead load due to asphalt wearing surface,
L = live load (static),
I = dynamic load,
R_n = resistance (load carrying capacity), and
ϕ = resistance factor.

In the selection of resistance factors, the acceptance criterion is closeness to the target value of the reliability index, β_T. The recommended resistance factors are ϕ = 1.00 for moment and shear in steel girders, ϕ = 0.90 for moment and shear in reinforced concrete T-beams, ϕ = 1.00 and 0.90 for moment and shear in prestressed concrete AASHTO-type girders, respectively.

Reliability indices calculated for bridges designed using the new LRFD AASHTO (ref. 1) are close to the target value of 3.5 for all materials and spans. The calculated load and resistance factors produce a uniform spectrum of reliability indices. For comparison, the ratio of die required load carrying capacity by the new LRFD AASHTO (ref. 1) and the AASHTO (ref. 8) varies from 0.9 to 1.2.

For OHBDC (ref.2), the load factors were not changed from the 1983 edition (ref. 9), but recommended resistance factor for prestressing steel is ϕ = 0.95. The resulting reliability indices are about 3.5.

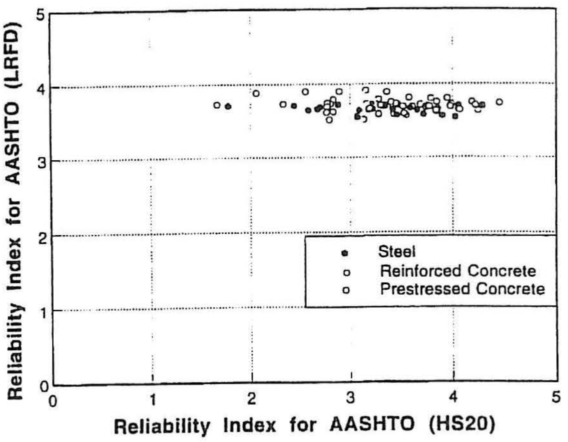

Figure 6. Reliability indices for AASHTO (1992) and LRFD AASHTO (1994) - Moment.

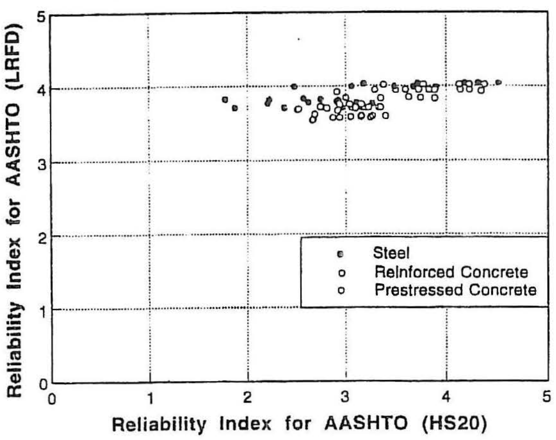

Figure 7. Reliability indices for AASHTO (1992) and LRFD AASHTO (1994) - Shear.

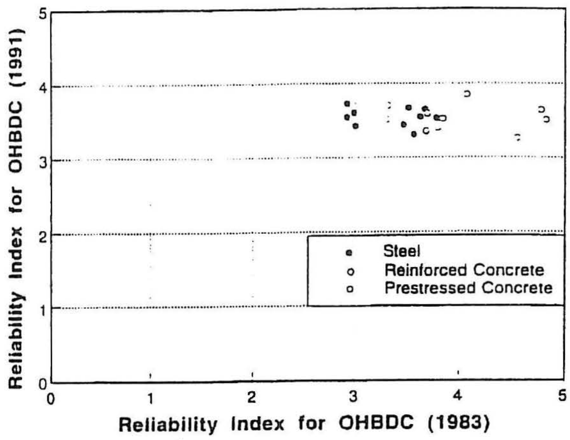

Figure 8. Reliability indices for OHBDC (1983) and OHBDC (1991) - Moment.

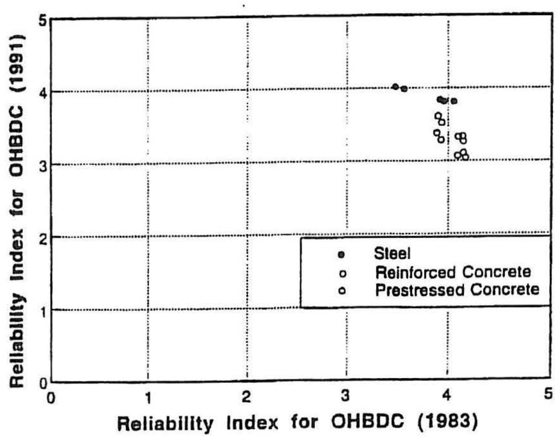

Figure 9. Reliability indices for OHBDC (1983) and OHBDC (1991) - Shear.

Conclusions

A brief introduction to the basic principles of structural reliability theory and its application to bridge engineering is given. Fundamental concepts like failure probability and reliability index are introduced. Ultimate as well as serviceability limit states for bridges are formulated, and as an example the reliability profile and a sensitivity analysis for a corroded reinforced concrete bridge are shown.

The calculated load and resistance factors in the calibration part of the paper provide a rational basis for the design of bridges. They also provide a basis for comparison of different materials and structural types. The study has several important implications. The calculated load and resistance factors provide a uniform safety level for various bridges. The statistical analysis of load and resistance models served as a basis for the development of more rational design criteria.

Bridge components designed using the proposed AASHTO LRFD (ref.1) and OBBDC (ref.2) have reliability index from 3.5 to 4.0, as shown in Fig. 6- 9.

Acknowledgements

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